EFFECT OF CAP BEAM TO COLUMN INERTIA RATIO ON TRANSVERSE SEISMIC RESPONSE OF MULTI COLUMN BRIDGE BENTS

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ABSTRACT :

In seismic design of bridge bents typically plastic deformations are allowed to occur at bridge column ends while the rest of bridge components remain essentially elastic due to maintenance and retrofit concerns. Based on observations of failures at past major earthquakes, CALTRANS bridge seismic code is structured in such a way that a ductile seismic response is preferred over a more rigid and brittle seismic response that targets to eliminate or minimize shear failures at bridge columns. Cap beam, expected to remain in essentially elastic range, can indirectly affect displacement ductility capacity of these bents. However, there exists considerable amount of examples of bridge bents having stumpy and rigid columns and flexible cap beam. In such a case, development of flexural plastic hinges takes place at cap beam and columns can experience shear failure before occurrence of flexural plastic hinges, showing a brittle mode of bridge failure. To investigate the consequences of such conditions, two different multi-column bridge bent examples are studied in transverse direction of bridge through pushover analysis. Bridge bent with a flexible cap beam, which is designed according to current design philosophy, has lower displacement ductility capacity due to the greater yield displacement capacity of bent. Pushover analysis showed that bridges having bents with stronger columns and weaker cap beam can display poor seismic behavior due to shear failure of columns or localizing inelastic region only at cap beam through formation of plastic hinges. Damaged cap beams can also risk unseating of superstructure.

KEYWORDS: bridge bent, seismic performance, displacement ductility

1. INTRODUCTION

Seismic performance of the multi span bridges, composed of column and cap beam bent system, is governed mainly by transverse bridge response. Structural damage on multi-column bents can occur in transverse direction due to seismic forces transferred from superstructure to substructure by shear keys. Whereas, in longitudinal direction, much less seismic forces are exerted on bent system compared to transverse direction. Abutments may be subjected to pounding due to longitudinal movement of superstructure in which superstructure can pound and stop at the end of a seismic event. It will be very hard to distinguish the cost of repair of abutment and bents after a seismic event but bridge bent repair costs can be significantly higher compared to abutment repairs at a multiple-span bridge. The current philosophy in the seismic design of bridge components is that each bridge component remains essentially in the elastic range experiencing no seismic damage except for column members due to maintenance and retrofit purposes. Plastic hinges are allowed to occur at column ends to dissipate the earthquake induced energy. Such design philosophy allows repairable seismic damage after a seismic event, which will not risk the use of bridge after the event Therefore, columns and consequently bridge bents should display a ductile behavior. New constructions of multi-column bridges can have stumpy columns having much greater moment of inertia than the cap beam. For this type of bridges, plastic hinges can develop at cap beams rather than columns and even columns can experience shear failure before occurrence of plastic hinges at columns. Moreover, damaged cap beams can cause seating problems for superstructure. The main objective of this research is to investigate effect of seismic load and displacement capacities, and associated inertia ratios of cap beam to column on transverse response of bridges. For this purpose, two bridge samples having different bent configurations are selected and pushover curves for both bents are developed. Bridge bents, designed according to the current design philosophy, have greater displacement ductility with less strength capacity. The other bridge bent with stumpy columns have higher strength capacity, but lower displacement ductility due to the occurrence of plastic hinges at the cap beam.

2. CURRENT DESIGN PHILOSOPHY

In Caltrans and ATC-32, capacity protected concrete components such as footings, bent cap beams, joints, and superstructure are designed to remain essentially elastic when the column reaches its overstrength capacity. By means of this condition, plastic deformations can be observed at the column members only, whereas the rest of the bridge components will contribute to the elastic deformations. In multi-column bents, if the bent is designed properly with respect to the current design philosophy, elastic displacement of the bent is calculated using the flexibilities of the column and the cap beam, whereas plastic displacement occurs in the columns only. As a result, displacement ductility capacity of the bent is reduced by the flexibility of the cap beam. This is illustrated by Priestley et al. (1996) comparing the rigid and flexible cap beam in pin supported bents as shown in figure 1.

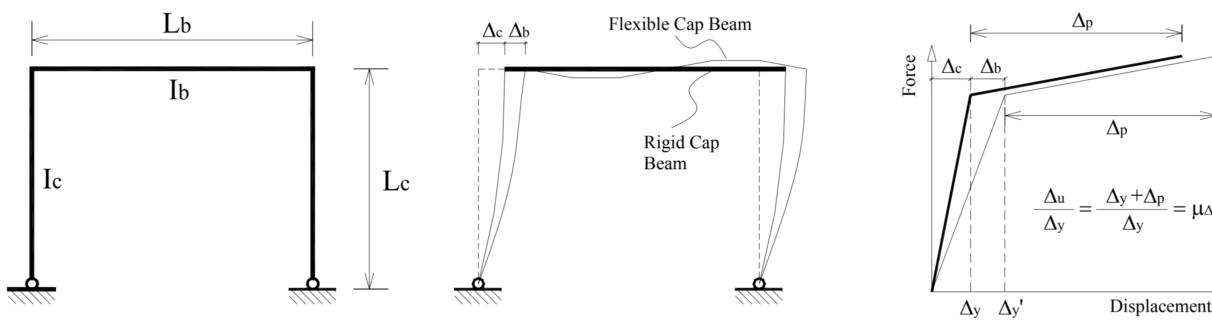

Figure 1 Increase in the elastic displacement of the bent due to the cap beam flexibility

Displacement ductility of the rigid and flexible cap beam bents are given as $\mu_{\Delta r}$ and $\mu_{\Delta f}$, respectively in Eqn. 2.1.

$$\mu_{\Delta r} = \frac{\Delta_y + \Delta_p}{\Delta_y} = 1 + \frac{\Delta_p}{\Delta_c}; \qquad \mu_{\Delta f} = \frac{\Delta_y' + \Delta_p}{\Delta_y'} = 1 + \frac{\Delta_p}{\Delta_c + \Delta_b} \tag{2.1}$$

Yield displacement of the two-column bent with flexible cap beam can be calculated using the column and cap beam flexibilities in terms of the yield displacement of the two-column bent rigid cap beam in Eqn. 2.2.

$$\Delta_y' = \Delta_y \cdot \left(1 + 0.5 \cdot \frac{I_c / L_c}{I_b / L_b}\right) \tag{2.2}$$

Displacement ductility of the bent with flexible cap beam is calculated using Eqn. 2.1 and Eqn. 2.2 as follows;

$$\mu_{\Delta f} = 1 + \frac{\Delta_p}{\Delta_y \cdot \left(1 + 0.5 \cdot \frac{I_c / L_c}{I_b / L_b}\right)} = 1 + \frac{\mu_{\Delta r} - 1}{1 + 0.5 \cdot \frac{I_c / L_c}{I_b / L_b}} \tag{2.3}$$

As can be seen in Eqn. 2.3., for a constant L_b/L_c ratio displacement ductility of the bent with flexible cap beam approaches to the displacement ductility of the bent with a rigid cap beam as cap beam inertia to column inertia ratio increases. This is also shown in figure 2 for L_b/L_c=1.0, which is the most frequent ratio among the newly constructed bridges in Turkey. L_b/L_c ratio distribution among the sample 55 bridges is shown in figure 3.

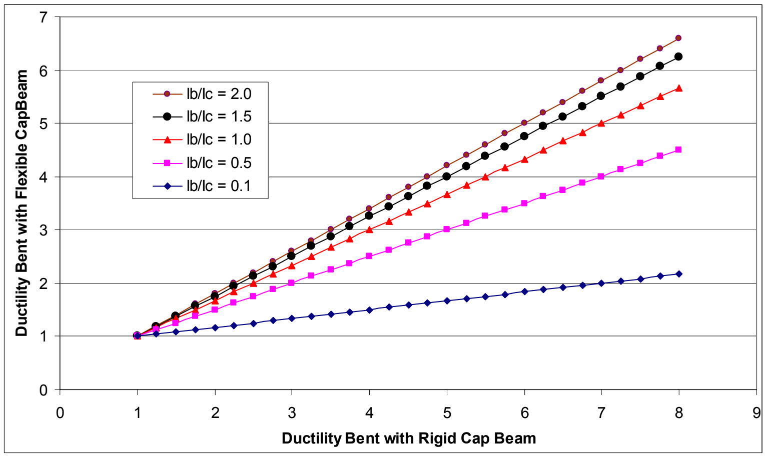

Figure 2 Ductility variation of bent for Lb/Lc = 1.0 for flexible cap beam wrt rigid cap beam

A representative ratio of 1.5 is given for the I_b/I_c ratio in ATC-32 for typical bridge bents. However, as can be seen in figure 3 (Avsar et al. 2006), most of the newly constructed bridges have a I_b/I_c ratio less than 1.0 in Turkey. The reason for the small I_b/I_c ratio is the stumpy and very rigid columns compared to the cap beams that are relatively flexible compared to the columns.

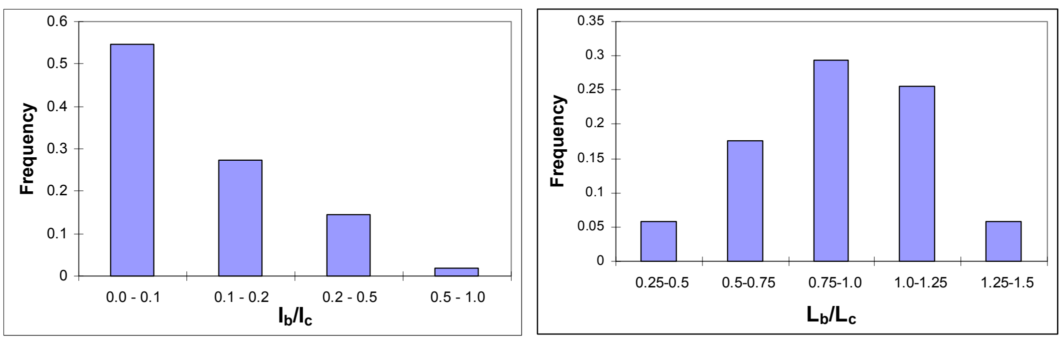

Figure 3 I_b/I_c and L_b/L_c ratio distributions among the newly constructed bridges in Turkey

3. SAMPLE BRIDGE BENTS

Two bridge systems having different bent configurations are considered in this study. The two bridges constructed at different years cross the same river and stay side by side. The old bridge as shown in figure 4, was constructed 30 years ago and it is still under service. Its superstructure is composed of 24cm thick RC slab and 4 RC beams having 0.45m width and 1.5m depth. It has a three column bent with a L_b/L_c ratio of 0.43 and I_b/I_c ratio of 4.97, indicating that its design is consistent with the current design philosophy. Since the cap beam is stronger than column, plastic hinges will be developed at the column ends rather than at the cap beams. The new bridge as shown in figure 5, was constructed 5 years ago. Its superstructure is composed of 22cm thick RC slab and 11 prestressed concrete T-beams. It has a two column bent with a L_b/L_c ratio of 1.13 and I_b/I_c ratio of 0.073. Since the cap beam is relatively weaker than the columns in the new bridge, plastic hinges are expected to develop at the cap beams first and then at the columns. Depending on the column length, column shear failure can occur before the development of plastic hinges at the cap beams. Since both bridges have the column length of 8m, column shear failure is unlikely to occur before the flexural failure of the members. Reinforce concrete components of the both bridge bents have the concrete and the reinforcement steel strength of 25 MPa and 420 MPa, respectively.

Figure 4 Old bridge with 3 column bent

Figure 5 New bridge with 2 column bent

4. CAPACITY CURVES OF THE BRIDGE BENTS

Capacities of both bridge bents in the transverse direction were calculated by pushover analyses. 2D bridge bent system was modeled in the OpenSees platform using the fiber sections for reinforced concrete members. Pushover curves as well as some important parameters giving necessary information about the damage state of the bent components are given in figure 6 and figure 7 for the old and the new bridge bents, respectively. These parameters are the reinforcement steel yield strain (ε_{sy}), the reinforcement steel fracture strain (ε_{su}), the strain at the peak concrete strength (ε_{co}), the ultimate unconfined concrete strain (ε_{cu}) and the ultimate confined concrete strain (ε_{ccu}). Since the number of columns and cap beams are very limited, redundancy in the bent is very low. Therefore, when the failure of any member takes place, which is defined as the first attainment of confined concrete strain to ε_{ccu} or the first attainment of reinforcement steel strain to ε_{su}, it is accepted that the bridge bent has reached its failure limit state. Final point for the pushover curves were rearranged considering the failure of the members. For the old bridge bent in figure 6, the ultimate displacement was calculated as 214.8mm when the column confined concrete strain has reached to its ultimate limit and column flexural failure has occurred. For the new bridge bent in figure 7, the ultimate displacement was calculated as 46.4mm when the cap beam confined concrete strain has reached to its ultimate strain.

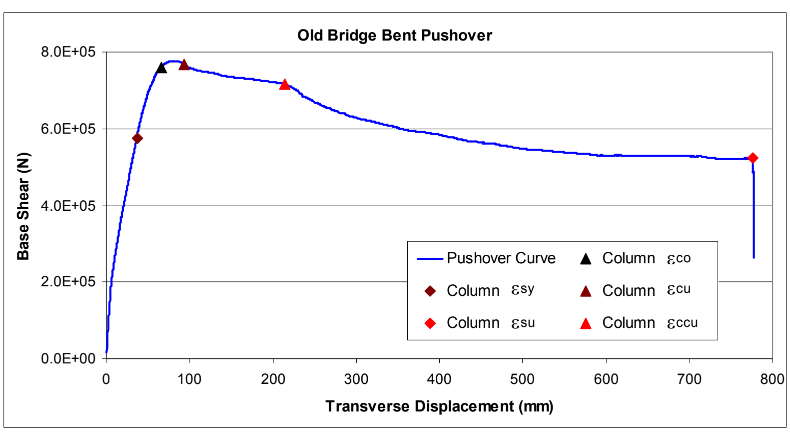

Figure 6 Pushover curve of the old bridge bent

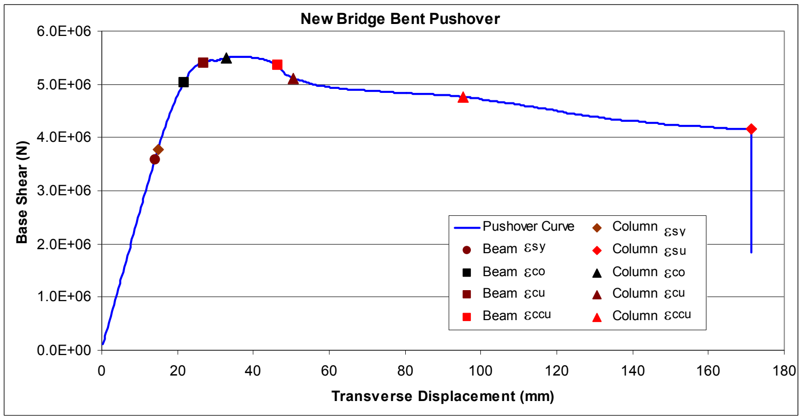

Figure 7 Pushover curve of the new bridge bent

In figure 6, at the old bridge that was designed according to the current design philosophy, plastic hinges were developed at the column ends and the cap beams remained elastic without experiencing any damage. However, cap beam of the new bridge bent has reached to inelastic limit state before the columns, which proves that the plastic hinges will be developed at the cap beam ends and they will not be in the elastic range any more if bridge bent has reached the yield limit due to the seismic actions.

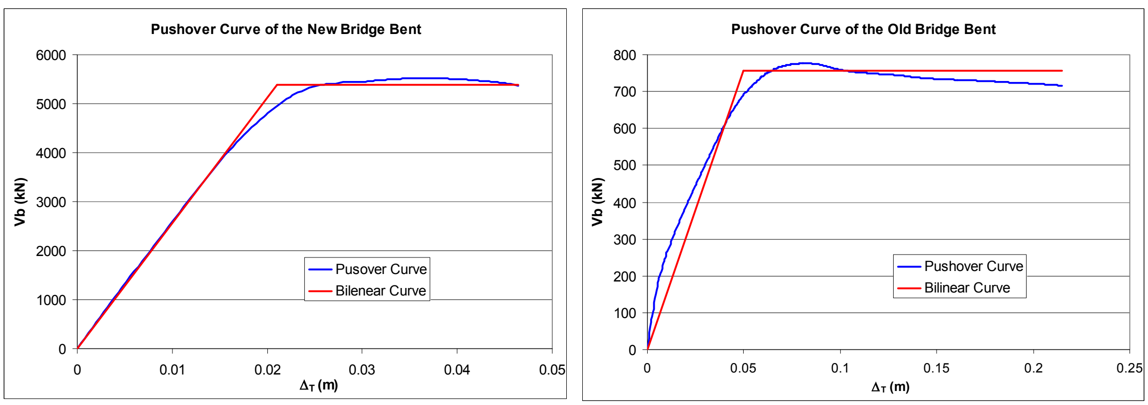

Figure 8 Pushover curve of the new bridge bent

The bilinear representations of the capacity curves (figure 8), widely used by other researchers, are constructed for both bridge bents in order to convert the capacity curves to the capacity spectrum. The capacity curve expresses overall shear force on all columns as a function of horizontal displacement of the bridge bent, whereas capacity spectrum represents the capacity curve in acceleration-displacement response spectra (ADRS) format. The spectral acceleration S_a and the spectral displacement S_d can be calculated using the modal parameters as shown in Eqn. 4.1 (ATC 1996) as presented in Table 1. Bridge bent system can be considered as a single degree of freedom system, and parameters of α and $\Gamma\Phi_N$ are calculated approximately as 1.0. In Table 1, it is shown that the new bridge bent is very stiff and has a very high strength compared to the old one. However, if the bridge bent goes beyond the elastic range, the old bridge bent has a displacement ductility capacity of twice as much as the new bridge bent. New bridge cap beam can experience excessive damage and can lead to progressive collapse.

Table 1 Basic parameters of the capacity curves and the capacity spectrum of bridge bents

Bridge Bent	$Sd_y=D_y$ (mm)	$Sd_u=D_u$ (mm)	Sa (g)	Fy (kN)	$K_{initial}$ (kN/m)	T (s)	M (ton)	Disp. Ductility Capacity, $\mu_{\Delta C}$
Old	49.8	214.8	0.449	756	15179	0.86	286	4.31
New	21.1	46.4	0.449	5393	256050	0.31	604	2.20

$$S_a = \frac{V_b/W}{\alpha}; \quad S_d = \frac{\Delta_T}{\Gamma\phi_N}$$

$$\alpha = \frac{\left[\sum_{i=1}^{N} m_i\phi_i\right]^2}{\left[\sum_{i=1}^{N} m_i\right]\left[\sum_{i=1}^{N} m_i\phi_i^2\right]}; \quad \Gamma = \frac{\sum_{i=1}^{N} m_i\phi_i}{\sum_{i=1}^{N} m_i\phi_i^2} \tag{4.1}$$

When the cap beams of the two sample bridge bents were considered to be infinitely rigid, displacement ductility capacities of the bents were recalculated by pushover analyses as 4.78 and 5.15 for the old and the new bridge bents, respectively. Considering the rigid cap beam bent ductility capacities ($\mu_{\Delta r}$), displacement ductility capacities ($\mu_{\Delta f}$) of the sample bridge bents with flexible cap beam were calculated using Eqn. 2.3. as 4.48 and 1.25 for the old and the new bridge bents, respectively. Although Eqn. 2.3. is derived for a two-column pin supported bent and for the elastic cap beams, a very reasonable ductility capacity was obtained for the old bridge bent. Since the cap beam to column inertia ratio is relatively high, ductility capacity of the old bridge bent with flexible cap beam is very close to the one for rigid cap beam. On the other hand, ductility capacity of the new bridge with flexible cap beam is very low compared to its rigid cap beam counterpart due to the occurrence of inelastic deformations at the cap beam and very low cap beam to column inertia ratio of 0.073.

5. SEISMIC DEMAND CALCULATION

Inelastic displacement ductility demands for the two sample bridge bents were calculated under the effect of the design response spectrum of the Turkish Earthquake Code 2006 and ten ground motions that were recorded in the three major earthquakes occurred in 90s in Turkey. The properties of these ground motion recordings with different scaling factors, the corresponding earthquakes and their displacement ductility demands from the sample bents are given in Table 2. In this study, foundation flexibility of the bridge bents was not taken into consideration. Therefore, a fully restrained boundary condition is assumed for the supports of the columns. In the light of this assumption, local site class of the bridge bents is taken as Z1 according to TEC2006 in order to calculate the design response spectrum. Design response spectrum of TEC2006, which is obtained for the 1st seismic zone (the highest), and the 5% damped response spectra of the earthquake recordings are compared in figure 9. Since the mean response spectra of the selected 10 ground motions satisfy the requirements of TEC2006 for the 475-year return period spectrum, these recordings were deemed to be appropriate for the calculation of inelastic deformations.

Table 2. Important features of earthquake records and their displacement demands from the sample bents

EQs	Station	Comp.	Scaling Factor	D* (km)	Site Class	PGA (g)	PGV (cm/s)	PGD (cm)	Old Bridge Bent ($\mu_{\Delta c}$=4.31) $D_{inelastic}$ (mm)	Old Bridge Bent Disp. Ductility Demand, $\mu_{\Delta d}$	New Bridge Bent ($\mu_{\Delta c}$=2.20) $D_{inelastic}$ (mm)	New Bridge Bent Disp. Ductility Demand, $\mu_{\Delta d}$
Kocaeli (08/1999, Mw7.4)	Sakarya	E-W	1.5	3.20	Rock	0.407	79.8	198.6	117.6	2.36	25.2	1.20
Kocaeli (08/1999, Mw7.4)	Izmit	E-W	1.5	4.26	Rock	0.227	54.3	129.3	68.8	1.38	27.3	1.30
Kocaeli (08/1999, Mw7.4)	Izmit	N-S	1.5	4.26	Rock	0.167	32.0	47.6	82.6	1.66	17.7	0.84
Kocaeli (08/1999, Mw7.4)	Düzce	E-W	1.5	17.06	Soil	0.383	46.6	108.6	222.8	4.47	38.9	1.85
Kocaeli (08/1999, Mw7.4)	Düzce	N-S	1.5	17.06	Soil	0.337	60.6	63.8	91.0	1.83	23.9	1.14
Düzce (11/1999, Mw7.2)	Düzce	E-W	1.0	8.23	Soil	0.513	86.1	170.1	126.3	2.54	26.0	1.23
Düzce (11/1999, Mw7.2)	Düzce	N-S	1.0	8.23	Soil	0.410	65.8	88.0	131.5	2.64	29.7	1.41
Düzce (11/1999, Mw7.2)	Bolu	E-W	1.2	20.41	Soil	0.821	66.9	21.3	234.6	4.71	26.8	1.27
Düzce (11/1999, Mw7.2)	Bolu	N-S	1.2	20.41	Soil	0.754	58.3	40.3	149.6	3.00	61.5	2.92
Erzincan (03/1992, Mw6.9)	Erzincan	E-W	1.0	2.00	Soil	0.469	92.1	58.1	139.8	2.81	27.0	1.28
D* :Closest distance to the fault rupture									average=	**2.74**	average=	**1.44**

The average displacement ductility demands presented in Table 2, are all lower than the ductility capacities of both bridge bents. Although the average demands appear to be similar for both bridge bents when compared to the ductility capacities, the response of each bent is quite different. It is worth mentioning that the significantly higher strength of the new bent does not seem to result in a favorable response. Under the given earthquake recordings, the old bridge bent reached its yield capacity and responded in the inelastic range experiencing various damage levels through formation of plastic hinges at the column ends. Displacement ductility demands for the new bridge bent are lower than the ones for the old bridge bent. Except for the N-S component of the Izmit, Kocaeli earthquake recording, the new bridge bent is in the inelastic range experiencing certain level of damage. When the new bridge bent reached its yield capacity, damage has initiated at the cap beams and then column damage occurred until the failure of the cap beam.

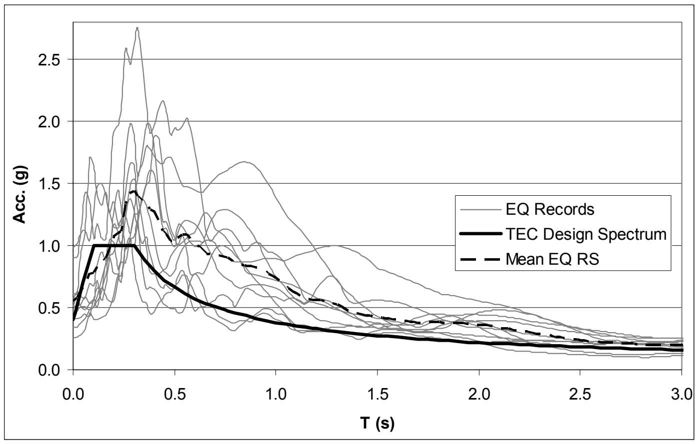

Figure 9 5% damped response spectra of the TEC2006 and the earthquake records

Among these 10 response history analyses, the results of the two components of the Bolu recording of the Düzce earthquake were investigated in detail. Under the effect of the N-S component of the Bolu recording, $\mu_{\Delta d}$=3.0 and $\mu_{\Delta d}$=2.92 are calculated for the old and the new bridge bent, respectively. In this case, $\mu_{\Delta c}$=2.20 of the new bridge bent is lower than the $\mu_{\Delta d}$=2.99 and failure occurs, whereas old bridge bent has sufficient displacement ductility capacity against the respective seismic demand. In the second case, under the effect of E-W component of the Bolu recording with a scaling factor of 1.2, it was calculated $\mu_{\Delta d}$=4.71 and $\mu_{\Delta d}$=1.27 for the old and the new bridge bent, respectively. As opposed to the first case, while new bridge has survived under the effect of this earthquake, the old bridge failed because its ductility capacity is lower than the ground motion's ductility demand. In the first case failure of the new bridge bent occurred due to the failure of the cap beam while in the second case column failure is the main reason for the failure of the old bridge bent.

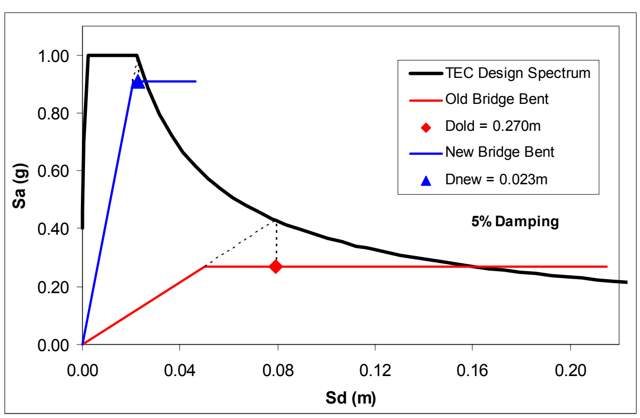

Figure 10 Performance point calculation for the 5% the ADRS of TEC2006

Performance points of the bridge bents were further calculated using the ADRS format of the design response spectrum of the TEC2006 and the capacity spectra of the bents as shown in figure 10. Corner period of the code response spectrum, which depends on the local site class of the bridge, is less than the fundamental period of the bridge bents in the transverse direction. Therefore, as mentioned in the TEC2006, inelastic displacements of the bridge bents were calculated according to the equal displacement rule. The performance points of the two sample bents in the transverse direction were calculated as 0.27m and 0.023m for the old and the new bridge bents, respectively. The corresponding displacement ductility demands of the code response spectrum are 1.59 and 1.08 for the old and the new bridge bents, respectively. Since the spectral accelerations of the mean response spectra of the 10 ground motions at the fundamental period of the bridge bents are higher than the ones for the design response spectrum, average displacement ductility demands of the ground motions are higher than the demands calculated by design response spectrum.

6. CONCLUSIONS

The effect of cap beam to column inertia ratio on the transverse response of multi column bridge bents has a considerable impact on the seismic behavior of the bridge. If the bridge was designed according to the current design philosophy, nonlinearity takes place only at the columns through formation of plastic hinges at the member ends and the rest of the bridge components remain essentially elastic, which is very beneficial and practical for the maintenance and retrofit purposes. In this case, ductility of the bridge bent is generally controlled by the column behavior and to a certain degree flexibility of the cap beam, which only contributes to the elastic deformation of the bridge bent, while all the plastic deformation takes place at the column members. Therefore, in order to provide a certain level of displacement ductility capacity of the bridges, yield deformation of the bridge bents are limited by designing stronger cap beams and weaker columns; provided that the plastic deformations occur only at the column ends and they have sufficient displacement ductility capacity.

Some examples of existing bridges, which were not designed according to the current design philosophy, have stumpy and rigid columns and flexible cap beams. In this case, plastic hinges initiate at the cap beams before the column reaches its yield capacity. Although, the strength capacity of such bridges is very high compared to the bridges with strong cap beams and weak columns, they have lower displacement ductility capacities. Therefore, failure of these bridge bents is unavoidable under the seismic effect of earthquake ground motions with high displacement ductility demands. On the other side, if the damage takes place at the cap beams over which the superstructure beams are placed, improper seating problem may occur or even unseating can take place.

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